# Design of Port@Lekki

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## ABSTRACT

*Port@Lekki (located about 70 km East of Lagos, Nigeria) is a port development for chemical tankers (160,000 DWT) and container vessels (8,000 TEU). The layout of the new port, including the layout of approach channel, turning circle and harbour basins has been derived from optimisations based on port operations, construction costs and possible future extensions. Two different breakwater concepts were applied for the main breakwater: A rubble mound with geo-bag core for the near-shore sections and a composite breakwater for the more exposed sections. The secondary breakwater was replaced by a barrier. The barrier consists of a core from sand, internally fortified by a protective geo-bag layer, a revetment on the harbour side and an artificial beach on the seaward side. The proposed layout of the new port and the design of main breakwater and barrier are described in this paper.*

## INTRODUCTION

The economy of Nigeria is rapidly developing in the past decade. The Port of Lagos, a main port to Nigeria, has reached the limits of its capacity; there is an urgent need for a new port in Nigeria.

Lekki Port LFTZ Enterprise (jointly promoted by Eurochem Corporation Pte Ltd., a Singapore based company and Lagos Free Trade Zone Enterprise) is planning to develop a Methanol plant as part of a petrochemical complex in the Lagos Free Trade Zone [LFTZ], Nigeria. As part of the LFTZ infrastructure Port@Lekki is planned; a deep water port for chemical tankers and container vessels. Port@Lekki will be the first “deep water” port in West Africa which can accommodate container vessels up to 8,000 TEU and liquid bulk vessels up to 160,000 DWT. Rambøll (L&T Rambøll Consulting Engineers Ltd., India) prepared a conceptual design for Lekki Harbour (including temporary, short term and long term development) which was further developed into the current layout plan by DMC.

Port@Lekki will be developed in two phases. In the first phase the port will include a Material Offloading Facility (for general cargo) and three container berths for ships up to 8,000 TEU and liquid berth for tankers of size 45,000 DWT and up to 160,000 DWT. The second phase consists of another liquid berth of 160,000 DWT and extension of Port towards the West. The main dimensions of the design vessels are specified in Table 1.

| Vessel | Size | Loaded displacement | LOA [m] | Beam [m] | Draft [m] |
|---|---|---|---|---|---|
| Liquid bulk carrier (Methanol tanker) | 160,000 DWT | 208,000 t | 310.00 | 47.00 | 17.10 |
| Container carrier | 8,000 TEU | 100,000 t | 335.00 | 45.60 | 14.50 |

**Table 1 Design vessel characteristics**

The design philosophy of the new port as well as the starting points and boundary conditions of the design are presented in this paper. The paper further describes the port layout and an unconventional concept for breakwaters and shore protection.

## PORT LAYOUT

The port layout, including the layout of approach channel, turning circle and harbour basins has been derived from optimisations based on port operations, construction costs and possible future extensions.

| Design vessel Size + type | Draft [m] | Design dredged depth (LAT) | | Block factor [-] | Width [m] |
|---|---|---|---|---|---|
| | | Inner channel [m] | Outer channel [m] | | |
| 8,000 TEU container | 14.5 | 16.0 | 17.0 | 0.8 | 150 |
| 160,000 DWT liquid bulk | 17.1 | 18.5 | 19.5 | 0.8 | 200 |

**Table 2 Approach channel dimensions**

The requirements for the two design vessels (8,000 TEU container ship and 160,000 DWT liquid bulk carrier) dictated the positioning of the container berths and of the liquid berths. The draft of the 160,000 DWT liquid bulk carrier is significantly larger then the draft of the container ship. Therefore the liquid bulk berths is located close to the port entrance. The liquid berth is located behind the main breakwater as shown in Figure 1.

The lower draft container berths are located inside the harbour basin, see Figure 1. The three container berths are in line at a straight quay wall of 1,538m length (including MOF). The container cranes can be shifted between the 3 berths and can be used flexible throughout the quay. The container berths are located along the north side of the harbour basin, with two berths located inside the actual basin and the third berth and the MOF located north of the turning circle. No berthing facilities have been foreseen at the south side of the harbour basin, however in the future this area could be developed. The location of the turning circle and harbour basin (partly seawards and partly landwards of the present shoreline) is based on an optimisation of the dredge and fill volumes. The orientation and positioning of the harbour basin is further determined by the requirements of the artificial beach that protects the harbour basin.

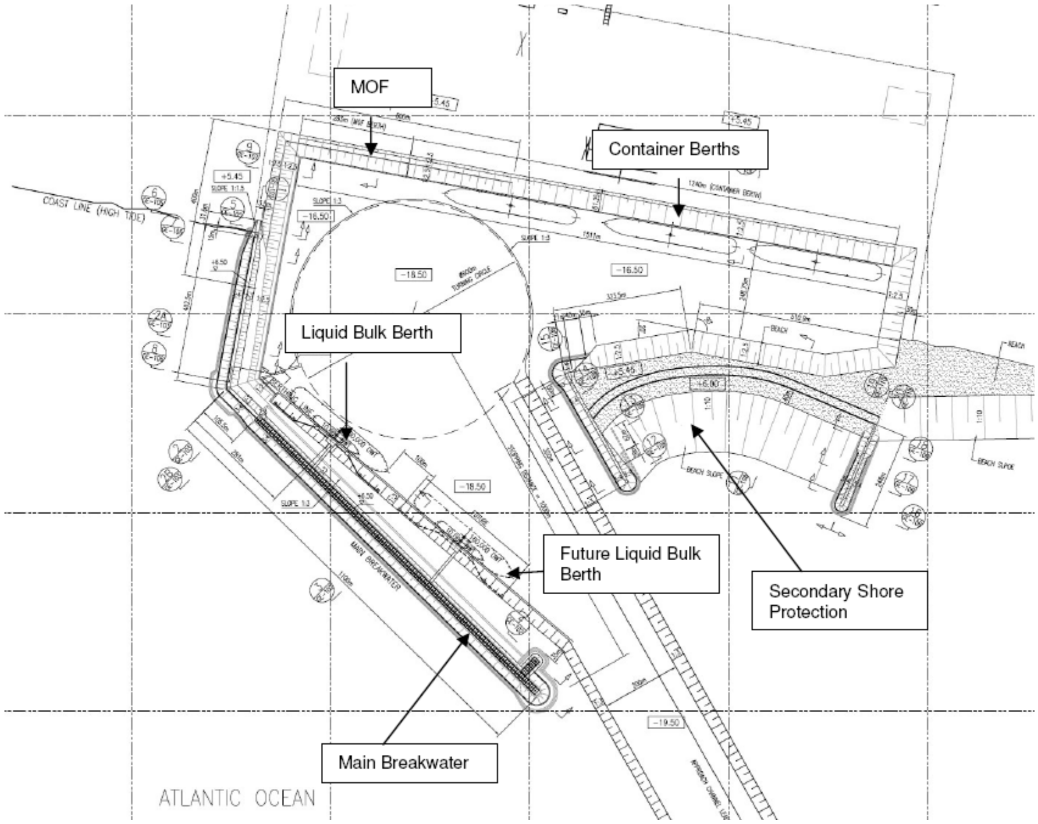


**Figure 1 Overview Port @ Lekki**

The main breakwater limits the wave penetration into the port and provides sufficient stopping distance in the shelter of the breakwater. These requirements result in a main breakwater with a length of 1,340m. A liquid berth will be located at the rear side of the main breakwater. The available space behind the main breakwater would allow for a second liquid berth (south-east of the first liquid berth) at a later stage. The port layout allows for future port expansion to the West of the turning basin or to the Northeast and/or East of the harbour basin.

The secondary breakwater is replaced by a barrier (located between harbour basin and sea), which consists of two rubble mound structures (groynes) and an artificial beach. The groyne next to the port entrance limits the wave penetration into the harbour and prevents sand transport from the artificial beach into the approach channel. The cross section of this groyne is similar to the nearshore sections of the main breakwater. The barrier (reclaimed land) consisting of a revetment on the harbour side and an artificial beach on the sea side. The beach is contained by two groynes (i.e. rubble mound structures). The barrier replaces a conventional rubble mound breakwater and was selected for the eastern part of Port@Lekki to limit the material take-off for rock and quarry run.

In the next sections the breakwater and the barrier are described in more detail.

**MAIN BREAKWATER**

The development of a concept for the main breakwater and the design verification in hydraulic model tests (conducted by DHI) are presented in this section. In Table 3 a summary of the Metocean design conditions is given.

| Return period | Offshore | | | Near-shore (LAT -10 m) | |
|---|---|---|---|---|---|
| | | | | Design high water level LAT +2.5 m | Design low water level LAT ±0.1 m |
| [years] | Significant wave height [m] | Peak wave period [s] | Wave direction [-] | Significant wave height [m] | Significant wave height [m] |
| 1:1 | 3.7 | 8 – 16 | S – SW | 4.5 | 4.1 |
| 1:10 | 4.1 | 9 – 16 | S – SW | 4.9 | 4.2 |
| 1:100 | 4.5 | 10 – 16 | S – SW | 5.1 | 4.4 |

**Table 3 Design METOCEAN conditions Port@Lekki (at depth contour LAT -10 m)**

Rubble mound breakwaters are applied frequently to create shelter for ports around the world. The advantage of a simple structure and cost efficiency at water depths up to 12 – 15m, the water depth at the breakwater of Port@Lekki is about 9.5m. Therefore, a rubble mound breakwater appeared to be a promising solution for Port@Lekki. However, the availability of rock material was a critical issue at this site. Access to the main breakwater was further required for piping and vehicles. This requires enough space on the breakwater crest and limited wave overtopping, leading to an even higher rock demand in the case of a rubble mound structure (see Figure 2a). To limit the rock demand alternative breakwater concepts have been considered.

The breakwater cross section has been further optimised (modified berm width, crestwall, geometry and rear slope) and the quarry run in the core of the breakwater has been replaced geotextile containers with sand fill (Figure 2b).

A composite breakwater has been considered to further reduce the core volume. By introducing a vertical structure at the centre of the breakwater the rock volumes would be reduced significantly. The vertical structure consisting of a row of piles was also tentatively considered, but was finally abandoned with respect to constructability (a.o. the problems related to pile driving with sea-based equipment in persistent swell).

It was concluded that a conventional caisson breakwater would not be cost efficient for this project with respect to the requirements for caisson fabrication (in a dry dock) and caisson installation (with persistent swell conditions at this site). The installation of the caissons in swell conditions would require support by a floating crane. The lifting capacity of the largest available crane was not sufficient for a caisson of reasonable length. Smaller caissons (with reduced caisson width and height) were not stable in a design storm. Therefore, these smaller caissons that could be efficiently fabricated and installed required additional protection by a rock slope. A composite structure was thus considered as an alternative for a rubble mound breakwater (Figure 2c).

Two different breakwater concepts were finally applied for the main breakwater; (i) a rubble mound with geo-bag core for the near-shore sections and (ii) a composite breakwater for the exposed sections (from bend to breakwater head).

The rubble mound with geo-bag core (extending from the bend to the shoreline) shall provide shelter for a material offloading facility. The caissons will be fabricated in the shelter of the rubble mound structure. In Figure 2 typical cross sections of rubble mound structures and composite structures are shown. The initial cross sections (Figure 2a and 2c) and the final cross sections including optimisation in physical model tests (Figure 2b and 2d) are presented.

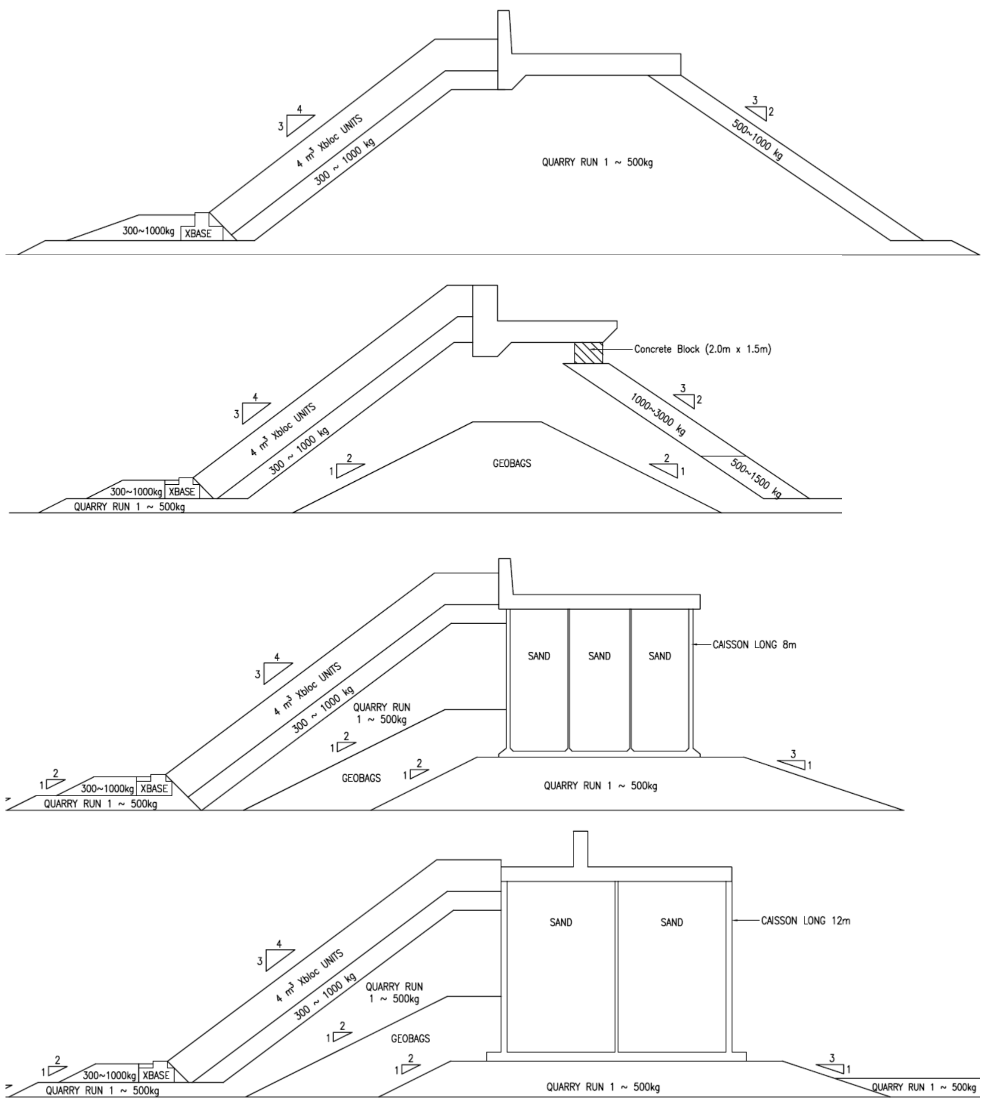


**Figure 2 Cross sections main breakwater: (a) Conventional rubble mound structure (top); (b) rubble mound structure with core from geotextile containers (2nd from top); (c) composite breakwater (2nd from bottom); (d) final layout of composite breakwater (bottom);**

The rubble mound structure consists of a geo-bag core, covered with a layer of quarry run (1 – 500 kg). The armour layer consists of Xbloc armour units (unit size 4 $m^3$). The crown wall on top of the rubble mound structure acts as wave screen, access road and support for piping.

The composite structure consists of a caisson protected on the seaward side by a rubble mound slope with a core from geo-bags covered with quarry run (1 – 500 kg). The armour layer of the rock slope consists of Xbloc armour units (unit size 4 $m^3$).


**Figure 3 Cross sections of main breakwater (2D model tests)**

The design of the main breakwater has been verified in hydraulic model tests. The model tests were performed by the Danish Hydraulic Institute (DHI), Copenhagen, Denmark. Cross sections of the rubble mound breakwater and of the composite breakwater were tested in a wave flume (2D), see Figure 3. The front slope of Xbloc armour units appeared very stable throughout the 2D test series; no damage occurred under design and overload conditions (20% larger waves than design waves). Design changes that have been derived from the results of 2D and 3D hydraulic model tests are described hereunder.

The rear side armour of the rubble mound structure was damaged by overtopping waves in the 2D model tests. The rear-slope was therefore shifted seawards in a way that the slab of the wave wall will protect the rear side armour against overtopping waves and will prevent unacceptable damage (see Figure 2b). The cross sectional area of the rubble mound (and thus the rock demand) has been reduced by this modification. The geometry of the berm was changeged (increased berm height and reduced berm width) to limit impact loads on the crest wall and to improve the stability of the crest wall. The thickness of the wave wall has been increased (to improve the overturning stability of the crest wall). The required width of the crest wall could be reduced to a minimum with these modifications of berm and wave wall.

The caisson of the composite breakwater (see Figure 3, right) was not stable in the 2D test series. The caisson was sliding under design and overload conditions. By shifting the wave wall on top of the caisson backwards, the horizontal loading on the caisson could be reduced (especially the impact loads of breaking waves). The downward pressure on the caisson deck (in front of the wall) further increased the caisson stability. The caisson with shifted wave wall proved to be stable.

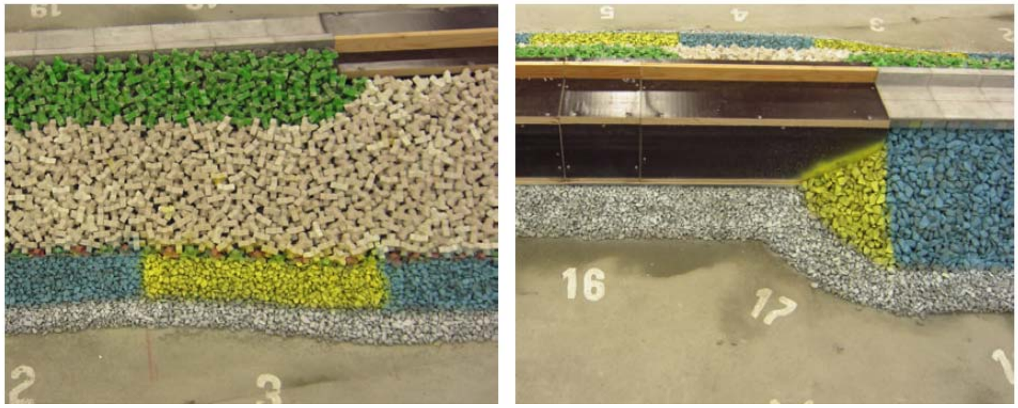


**Figure 4 Transition zone between caisson and rubble mound breakwater**

The breakwater head and the transition between composite breakwater and rubble mound breakwater were tested in a wave basin (3D), see Figure 4. During the 3D model tests the incoming waves were diffracted at the breakwater head and refracted at the slopes of the approach channel. The wave heights in the lee of the breakwater due to the combined effect of swell (with wave periods of 18 s), wave induced flow around the roundhead and wave refraction at the slopes of the approach channel were significantly higher than predicted by common engineering tools (i.e. Boussinesq wave model or diffraction diagrams).

The rear side armour stability of the rubble mound section was also investigated in the 3D tests. The rock armour on the rear slope experienced damage levels from 3.8% at the transition from rubble mound to caisson to more than 15% in the following sections of the rubble mound breakwater. This damage resulted from the combined effect of wave forces exerted to the stones by overtopping and by waves agitation at the lee side.


**Figure 5 Spur breakwater (altered rear side armour layer circled)**

A spur breakwater was introduced close to the breakwater head to provide shelter to the rear side of the breakwater, see Figure 5. The spur breakwater has a length of 48m and will reduce the wave heights at the port entrance from about 60 – 70% of the incoming waves to less than 40%. The rock size on the rear slope at the transition from rubble mound to composite breakwater has been increased to 1-3 ton.

## BARRIER (PROTECTIVE BEACH)

The barrier concept (including the design of the protective beach) is presented in this section.

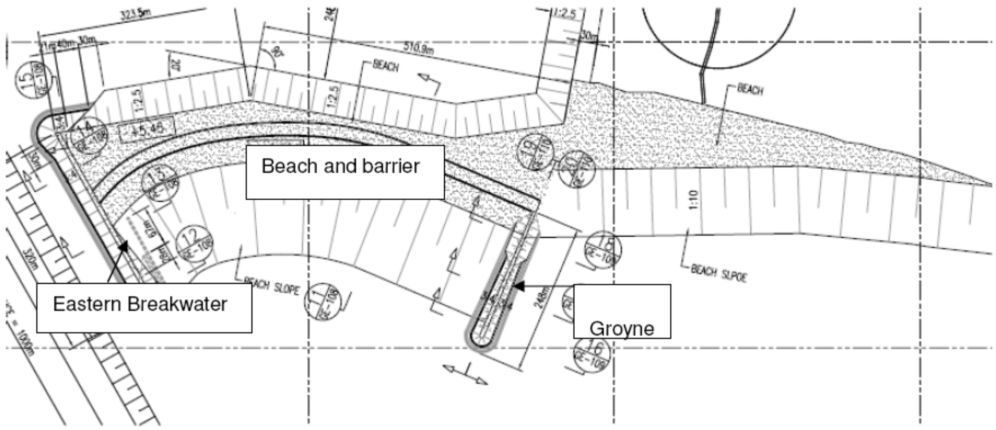


**Figure 6 Eastern breakwater, beach and groyne**

The barrier concept was selected with respect to the environmental conditions (i.e. little variation in wave directions), availability of construction materials and constructability, the barrier layout is presented in Figure 6. The availability of rock material is critical at this site. A soft shore protection was therefore proposed to protect the harbour basin. A land reclamation with an artificial beach (contained by 2 groynes) is foreseen as a replacement for a conventional rubble mound breakwater. The advantage of this barrier concept is twofold, (i) dredged material can be used efficiently and (ii) the amount of rock material is reduced. However, a soft shore protection requires some protection, because cross shore sediment transport during a severe storm may erode the protective beach. A protective layer inside the reclaimed sand body of the barrier is therefore foreseen (Figure 7).

The protective layer strengthens the barrier and will prevent breaching of the barrier during a design storm. The protective layer will contain the sand barrier temporarily when the beach profile is eroded and shall withstand design storm conditions for several hours. The barrier consists of a core from sand, internally fortified by a protective geo-bag layer, a revetment on the harbour side and an artificial beach on the seaward side.

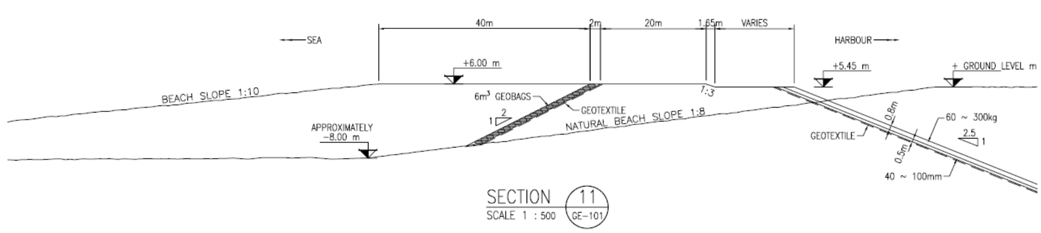


**Figure 7 Cross section barrier**

Next to the entrance channel a groyne (eastern breakwater) is foreseen to limit the wave penetration into the port and to prevent sand transport from the artificial beach into the approach channel. The layout of this groyne will be similar to the nearshore sections of the main breakwater. The seabed level at the head of the eastern breakwater is at -9 m to -10 m LAT, the groyne is partially sheltered by the main breakwater.

The shape of the artificial beach (i.e. beach profile and orientation) are mostly determined by the prevailing angle of wave approach. The beach is parly sheltered by the main breakwater; the local wave directions along the seaward face of the barrier were analysed by a numerical wave model (Boussinesq model). An example of the wave pattern at the harbour entrance and at the barrier is presented in Figure 8. The shape of the artificial beach was derived from the static equilibrium of a crenulate shaped beach according to the parabolic model of Hsu and Evans (1989). The concluded equilibrium beach was checked against the results of the numerical modelling (i.e. near-shore wave directions along the proposed beach). It was further assumed that a cross-shore beach profile that is similar to the natural beach slope (approximatele a 1:8 slope) will be in equilibrium. A 1:10 slope is tentatively proposed for the artificial beach. The shape of the artificial beach will vary in time. Some erosion can be expected during the summer; during winter when the wave heights are generally smaller the beach will be (partly) restored. The artificial beach might require some maintenance. However, the combined effect of groynes (at the up-wind and down-wind end of the bach) and a beach shape that is close to equilibrium will limit the maintenance requirements to a minimum.

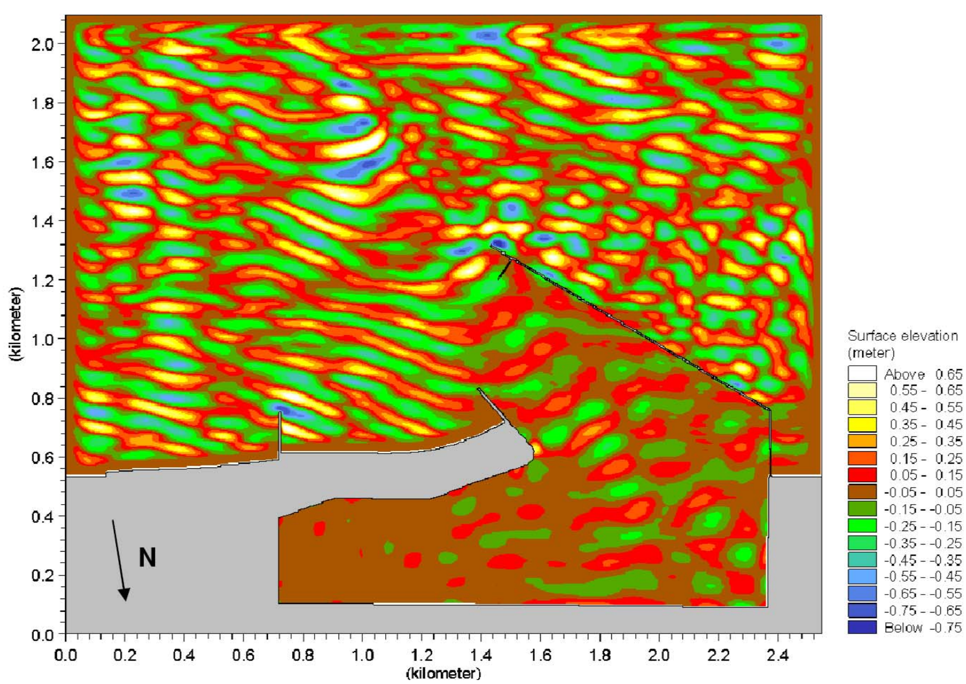


**Figure 8 Wave model, diffraction around main breakwater**

## LESSONS LEARNED

The design of Port@Lekki was strongly driven by the search for a cost efficient harbour layout. This resulted in unconventional concepts for main breakwater and secondary breakwater. The optimisation of the marine structures during the model testing provided useful insight into the hydraulic performance and stability of these structures. The lessons that have been learned from this project are briefly described in this section.

*Design vessels:* Port owners tend to be optimistic with respect to the design vessels and may specify design vessels that will not visit the port during the first years of operation. As a result the berths and quay walls as well as the dredged depth of approach channel and harbour will be overdesigned with respect to the requirements of the first years. It might be more cost efficient to let the port grow with the increasing vessel size. This requires a flexible layout of the port, which meets the present requirements and can be extended with limited effort to accommodate larger vessels. The cost implications of large design vessel and the requirements of a flexible port layout should be analysed in an early stage of the project and need to be communicated to the port owner.

*Berth arrangement:* The positioning of the berths inside the port is mostly driven by operational aspects. However, the berth arrangement may also affect the construction costs of a port. At Port@Lekki the berths for the larger draft vessels (liquid bulk carriers) are located close to the port entrance and the container berth are located in the harbour basin. This arrangement is favourable with respect to dredging volumes (required depth of harbour basin is reduced by 2.5 m) and with respect to operability (container berths are more sheltered than liquid bulk berth).

*Survey data:* The interpretation of bathymetric survey data requires special attention. Discrepancies were found between land based and sea based topographic surveys. Different reference systems had been used for the surveys leading not only to a horizontal shift but also to a vertical shift. The land based survey was based on the local Nigerian Mina datum whereas the marine survey was based on the international WGS84 datum. A careful review of survey data is a must for port engineers.

*Model testing:* The combined effect of wave diffraction and wave energy focussing of the entrance channel caused unexpected large wave heights at the port entrance and in the lee of the main breakwater. These waves have damaged the rear armour of the breakwater and would have also an adverse effect on navigation. The wave propagation into the port has been efficiently reduced by a spur breakwater of less than 50m length. This problem was not foreseen during the initial design; it underlines the importance of model testing and painfully illuminates the shortcomings of the engineering practice.

## REFERENCES

Hsu, J.R.C., and C. Evans, 1989. Parabolic bay shapes and applications. Proc., Institution of Civil Engineers, London, England, Vol. 87 (Part 2), 556 - 570.